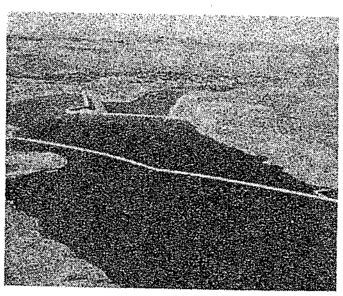
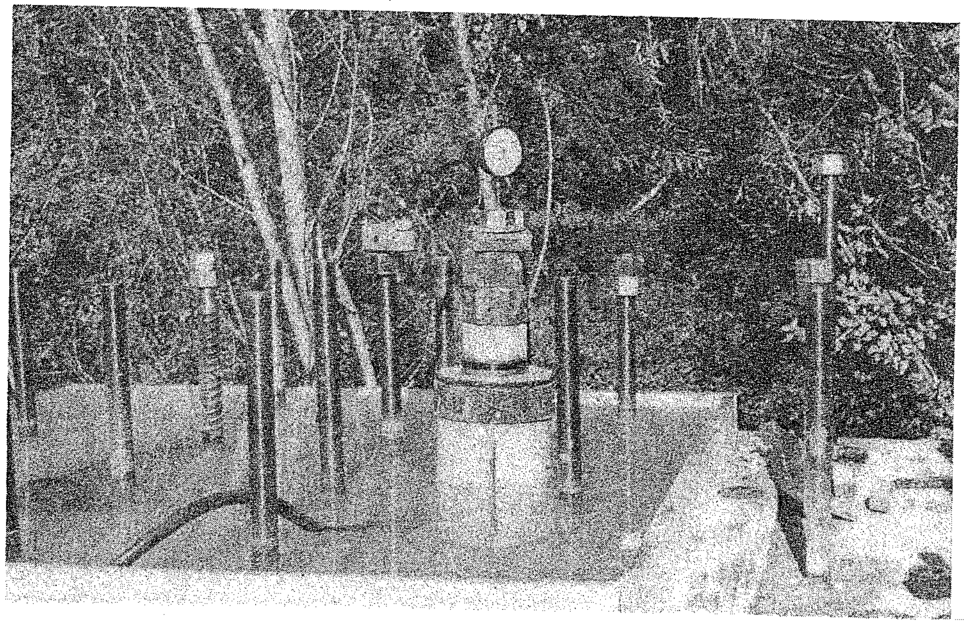


The REMR Bulletin

News from the Repair, Evaluation, Maintenance, and Rehabilitation Research Program

VOL 5, NO. 2 INFORMATION EXCHANGE BULLETIN JUL 1988



Evaluation of Vinylester Resin for Anchor Embedment in Concrete

by
Jim McDonald
US Army Engineer Waterways Experiment Station

The Old River Low Sill Control Structure, located on the west bank of the Mississippi River about 40 miles southwest of Natchez, Mississippi, was completed in 1960. After 16 years in operation, the stilling basin had suffered abrasion-erosion damage to the extent that repair was required to protect the integrity of the overall structure. Thirty prefabricated modules of 1/2-in.-thick steel plate, 24 ft long and from 3 to 22 ft wide, were anchored to the top of the end sill and to the floor slab (Reference 1). The anchors were embedded with prepackaged polyester resin grout. Repairs were completed underwater with careful use of partial gate closures to produce acceptable working conditions.

An underwater inspection of the repairs in 1977 revealed that seven of the modules had suffered at least partial loss of steel plate and that some of the anchors had been pulled completely out of the concrete (Reference 2). Subsequent inspections revealed progressive damage to the modules until practically all of the steel plate had been lost. In addition, a diver inspection in 1986 revealed damage to the gate guide rail system in the three low bays (Reference 3). Consequently, the features required to install the stop-log closure were inspected, and damage to the needle-seat recess was discovered.

The planned dewatering of the stilling basin for inspection and repair in 1987 necessitated the repair of the needle-seat recess and the guide-rail system. Since the repair of the needle-seat recess (Figure 1) would have to be done underwater, there was some concern as to the appropriate grout for embedding the anchors into the existing concrete. The performance of polyester resin grouted anchors in the stilling basin and results of laboratory tests (Reference 4) caused the District to be reluctant to specify polyester resin for additional underwater repairs.

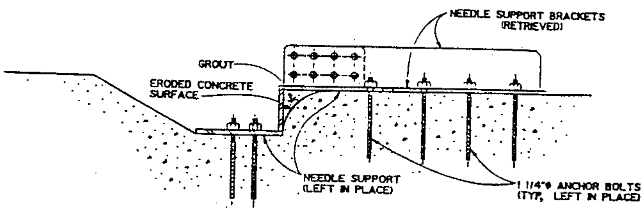


Figure 1. Needle-seat recess repair.

A review of available manufacturers' literature on concrete grouting systems revealed that Hilti, Inc., was promoting HEA vinylester resin adhesive as "the optimal solution for heavy duty fastenings in dry, wet, and temperature-stressed base materials." According to the manufacturer's representatives, anchors embedded in vinylester resin under submerged conditions performed as well as comparable anchors installed in the dry. However, no test data were furnished to substantiate this claim. Therefore, the New Orleans District requested that the Waterways Experiment Station initiate a study to evaluate the load-carrying capacity of anchors embedded in concrete with vinylester resin grout.

ANCHOR INSTALLATION

Vertical holes to depths of 15 in. were drilled in a mass concrete block with a 1-1/2-in. outside-diameter core barrel. After the concrete cores were removed, half of the holes were filled with turbid water from the Mississippi River near Vicksburg, Mississippi. Drilling water in the remainder of the holes was removed with pressurized air, and the holes were allowed to dry for a minimum of 3 days before anchors were installed.

Two types of anchors, high-strength threaded steel rods (ASTM A 193 Grade B-7) and reinforcing steel bars (Grade 60), were used. Both types of anchors were 1-1/4 in. in diameter and 30 in. long. One end of each anchor had a flat chisel point, and the opposite end of the reinforcing bar was threaded for approximately 4 in.

Hilti personnel directed the installation procedure. Two sizes of glass capsules (1- by 8-1/4-in. and 1-1/4- by 12-in.), each containing quartz sand, benzol peroxide hardening agent, and vinylester resin, were used in each hole. The larger capsule was placed into the bottom of the drill holes and crushed by repeatedly stabbing with the chisel-point end of the anchor. A smaller capsule was then placed into the drill hole. The vial was crushed, and immediately the anchor was spun into the hole with an electric drill. The resin extruded from the dry holes was very cohesive, a fact that may account for the significant effort required to attain the full embedment depth.

A similar procedure was used to install the anchors under submerged conditions. However, anchor installation under submerged conditions required significantly less effort. Dry-hole installation required approximately 75 sec; underwater, approximately 45 sec. Also, under wet conditions, the extruded grout was much more fluid, and its creamy color contrasted with the black grout extruded under dry conditions. It appeared that the turbid water actually mixed with the vinylester resin during the anchor installation process.

TESTING

A hollow-core hydraulic ram and an electrically powered hydraulic pump were used to load the anchors in both the tensile and shear tests. A universal laboratory testing machine was used to calibrate the loading system.

In the pullout tests, the hydraulic ram was positioned over the anchor and secured with a nut threaded onto the end of the anchor (Figure 2). A mechanical dial gage positioned on the end of the anchor measured displacement of the anchor relative to the concrete surface.

Horizontal shear loads were applied to the anchors through a doughnut-shaped steel collar positioned around the anchor (Figure 3). A high-strength steel rod was used to transfer load from the hydraulic ram through the collar to the anchor. Grouted anchors were used to mount a reaction beam on the sides of the concrete block.

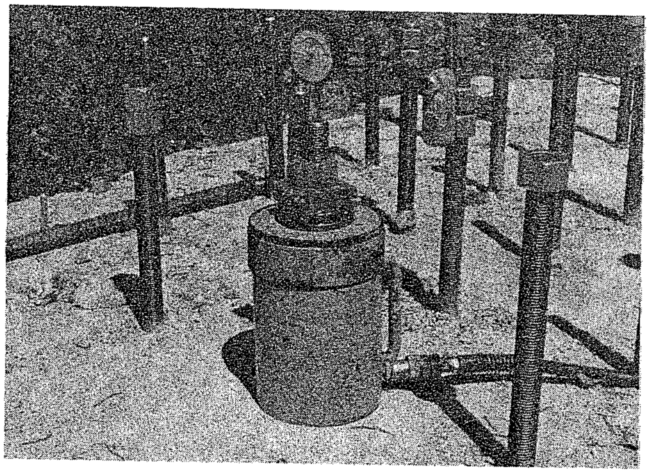


Figure 2. Pullout tests in progress.

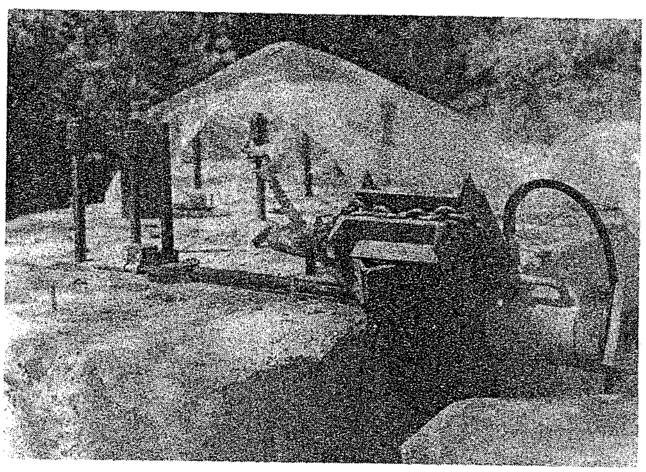


Figure 3. Arrangement of shear test equipment.

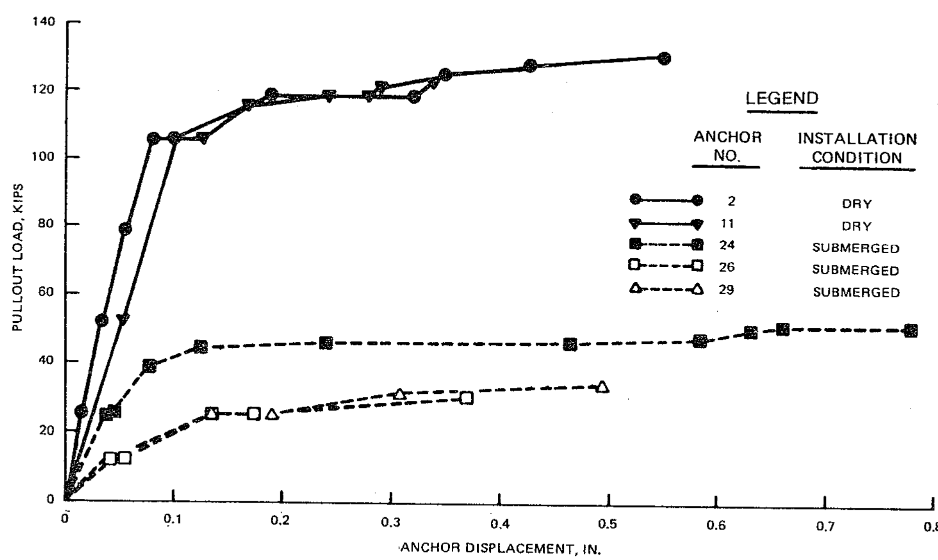
An LVDT gage positioned against the collar opposite the load-transfer rod monitored horizontal movement of the anchor near the surface of the concrete block.

PULLOUT TEST RESULTS

Results of pullout tests conducted at 1-day age are shown in Figure 4. Anchors embedded in dry holes exhibited small displacements at loads to approximately 105,000 lb force (105 kips). However, beyond this load the anchors exhibited significant displacement with relatively small increases in applied tensile load. In comparison, anchors embedded under submerged conditions did not exhibit bilinear load-displacement curves, making it difficult to determine precisely the load at which bond failure occurred at the grout-concrete interface. Therefore, pullout loads at displacements of 0.1 and 0.2 in., in addition to the ultimate load, were selected as a basis for comparison of anchor performance under the various installation conditions. For example, at 0.1-in. displacement the average tensile capacity of anchors installed under submerged conditions was 27.1 kips, approximately one-fourth that of similar anchors installed under dry conditions.

An inspection of the anchors after testing revealed that failure occurred through loss of bond at the grout-concrete interface. This bond loss was especially evident for anchors installed under submerged conditions. To determine the cause of the relatively poor performance, these anchors were pulled completely out of the drill holes so that the vinylester grout could be inspected. The grout was very soft and was easily removed from the anchors by hand.

Figure 4. Results of pullout tests conducted at 1-day age.



Test results at 3, 7, and 28 days on anchors installed in dry holes were similar to the results at 1-day age. In comparison, tests on anchors installed under submerged conditions yielded relatively erratic results with pullout loads ranging from 25.4 to 55.7 kips at 0.1-in. displacement (Figure 5).

Attempts to improve the performance of anchors installed under submerged conditions by cleaning the drill holes prior to anchor installation were not successful. The average tensile capacity of anchors installed in holes flushed with tap water was only 7 percent higher than that of anchors installed in as-drilled holes containing turbid water. Additional cleaning of the holes with a bristle brush during the flushing process failed to improve anchor performance (Figure 6).

A limited number of tests were conducted to evaluate the performance of No. 10 reinforcing bars as anchors. Results indicate that ultimate tensile capacities are essentially the same for comparable threaded-rod and reinforcing-bar anchors.

SHEAR TEST RESULTS

Some bending of the anchors occurred during the shear tests as a result of localized failure of the grout and concrete near the top of the holes. Although this bending probably caused some error in the test data, the results are considered a satisfactory estimate of the relative shear capacity of anchors installed under the various conditions.

Shear loads at displacements of 0.2 and 0.4 in., in addition to the ultimate load, were selected as a basis for comparison of anchor performance under the various installation conditions. At displacements of 0.2 and 0.4 in., the average shear load was slightly higher for anchors installed in as-drilled holes under submerged conditions than for those installed in dry holes (Figure 7). If the anchors installed under submerged conditions had been loaded to failure, their ultimate shear capacity could have followed this trend. However, inadequate reaction beams necessitated stopping the

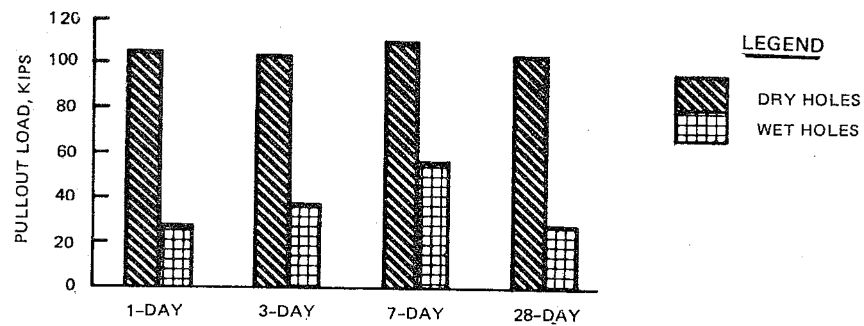


Figure 5. Summary of pullout test results at 0.1-in. displacement.

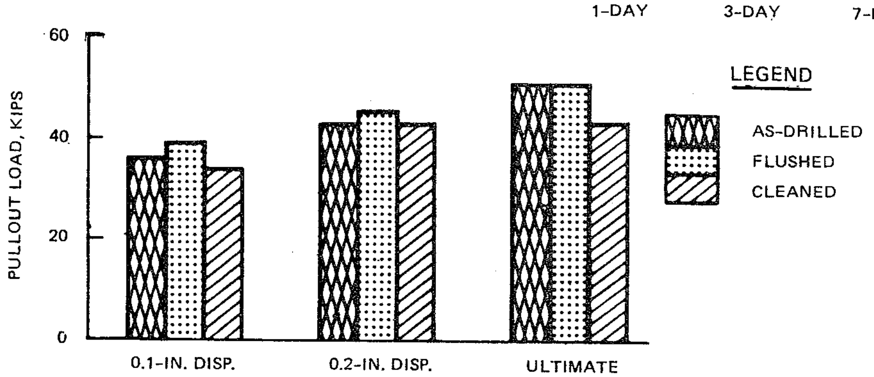


Figure 6. Summary of pullout test results for anchors installed under submerged conditions in holes of varying cleanliness.

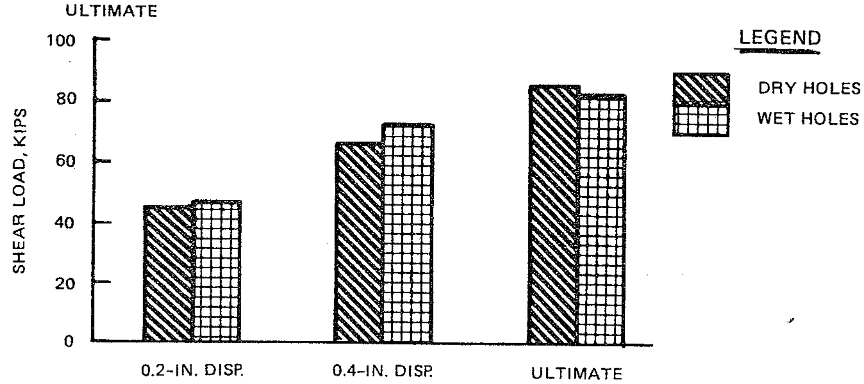


Figure 7. Summary of shear test results.

tests at an average shear load slightly less than the ultimate load for anchors installed in the dry. Similar results were obtained in tests conducted at 9 days age.

Overall, ultimate shear capacities ranged from 73.5 to 93.2 kips with an average of 82.2 kips. Excluding the results of two tests, the upper and lower bound values, ultimate shear capacities ranged from 77.0 to 85.6 kips with an average of 81.9 kips. Accordingly, ultimate shear capacities were all within 10 percent of the average, regardless of installation condition and testing age. This result is attributed to the relatively small annulus present when a 1-1/4-in.-diameter anchor is embedded in a hole drilled with a 1-1/2-in.-diameter core drill. In essence, the test results reflect the shear capacity of the steel anchor with primary resistance being provided by the concrete with little, if any, contribution by the embedment material.

CONCLUSIONS

For the range of parameters in this study (hole condition and test age), results of pullout tests on threaded-rod anchors installed in dry holes were remarkably consistent with an overall average tensile capacity of 105 kips at 0.1-in. displacement and an average ultimate load of approximately 125 kips. In comparison, results of pullout tests on anchors installed under submerged conditions were relatively erratic with an overall average tensile capacity of 36 kips at 0.1-in. displacement and an average ultimate load of 48 kips. Obviously, the tensile load capacity of anchors embedded in concrete with Hilti's HEA vinylester resin capsules is significantly reduced when the anchors are installed under submerged conditions. At a displacement of 0.1 in., the tensile capacity of anchors embedded under submerged conditions was approximately one-third that of similar anchors embedded in dry holes.

The significantly reduced tensile load capacity of anchors embedded in concrete with HEA vinylester resin capsules under submerged conditions should be recognized in any design of anchor systems for underwater applications. For the types of anchors and installation conditions described herein, a maximum tensile load of not more than 24 kips is recommended for design of underwater anchor systems subjected to short duration loads. This load was determined by reducing the overall average tensile capacity at 0.1-in. displacement by the standard deviation. Appropriate factors of safety should be used to calculate the maximum allowable tensile load.

Creep tests should be conducted to evaluate the effect of sustained loads on anchor performance prior to the use of HEA vinylester resin capsules for embedment of anchors that will be subjected to long-term loads.

For further information, contact Jim McDonald at (601) 634-3230.

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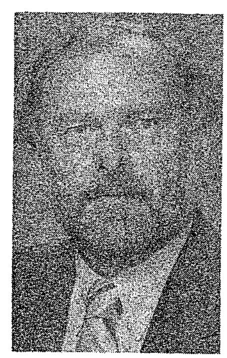
Jim E. McDonald is a research civil engineer in the Concrete Technology Division, Structures Laboratory, Waterways Experiment Station. He is Problem Area Leader for the Concrete and Steel Structures portion of REMR and is also principal investigator for four REMR work units, including 32303, "Application of New Technology to Maintenance and Minor Repair." He has been involved with various aspects of concrete research for 27 years and has authored more than 70 technical reports and articles. McDonald received his B.S. and M.S. degrees in civil engineering from Mississippi State University.



WORKSHOP ON REPAIR AND MAINTENANCE OF SHALLOW-DRAFT TRAINING STRUCTURES VIDEO AVAILABLE

"Repair and Maintenance of Shallow-Draft Training Structures" was the topic of a REMR-sponsored workshop held February 24-25, 1987, at the US Army Engineer Waterways Experiment Station, Vicksburg, Mississippi. Over 45 individuals were in attendance, representing 13 Corps of Engineers Divisions and Districts. Each District gave a presentation detailing its current dike-repair work load and typical methods of repair and evaluation used.

To request a videotape, call (601) 634-2651 or 4146, or write to: Commander and Director, US Army Engineer Waterways Experiment Station, ATTN: CEWES-HR-P/Mr. Dave Derrick, PO Box 631, Vicksburg, MS 39180-0631.

Speaker	*Topic*	*Time*	*Tape Counter*
Dave Derrick	Introduction	0:08	0000
Dick Sager	Welcome	3:20	0020
Bill McCleese	Opening Remarks	14:04	0199
Bob Athow	WES REMR Repair Techniques	11:00	0798
	Presentations from the Districts		
Mike Trawle, WES		11:22	1239
Laura Broderick, Portland District		19:00	1611
Claude Strauser, St. Louis District		15:42	2180
Danny Hare, Rock Island District		8:08	2594
Tom Burke, Kansas City District		24:32	2798
Andy Lowery, Memphis District		16:06	3360
Robert Young, Little Rock District		14:19	3704
Dennis Johnson, Tulsa District		15:40	3995
Steve Earl, Omaha District		10:30	4303
T. K. Grant, Vicksburg District		17:47	4504
Ken Wrightman, St. Paul District		13:36	4834
Jim Pennington, WES		16:54	5082
	Total Time:	3:32:08	

WORKSHOP ON MANAGEMENT OF BIRD PESTS

The US Army Corps of Engineers sponsored a workshop on the management of bird pests on April 27-29, 1988, in New Orleans. Seven bird damage control experts from the Animal Damage Control Section of the Department of Agriculture and an ecologist from USA-CERL presented nine papers and two panel discussions dealing with bird management strategies. The speakers represented a good balance of wildlife managers, bird control program administrators, and researchers. The bird species discussed included all common pests at Civil Works Projects: pigeons, starlings, house sparrows, blackbirds, Canada geese, and gulls. All known bird control techniques and technologies were thoroughly discussed. Potential environmental impacts, ecological implications, toxicity hazards, and public reactions associated with bird control were also addressed. An important aspect of the workshop was the outstanding participation and enthusiastic discussions among all attendees and speakers. Attendees expressed an interest in an annual workshop and made suggestions to include pests other than birds and to open the workshop to all Department of Defense personnel. Abstracts of the presentations along with additional material will be published as a REMR technical report.

Floating Debris Boom Evaluation Program Summary

by
Roscoe E. Perham
US Army Cold Regions Research and Engineering Laboratory

The floating debris boom at Chief Joseph Dam on the Columbia River, Bridgeport, Washington, was instrumented for forces and associated physical measurements such as wave heights and pressures and wind velocity and direction. The 3,000-ft-long flexible boom was built from surplus 6-ft-diameter by 12-ft-long Navy buoys, wire ropes, fittings, and connections and was designed to have great strength. Initially, the Seattle District had requested funding to have the measurements made because a sampling of the boom forces with a load cell in the left bank anchor, at the downstream end of the boom, indicated that the forces were very low (in the range of 1 to 2 tons) for something using 2-1/2-in.-diameter cable.

In response to an unsolicited proposal from the University of Washington Civil Engineering Department, the instrumentation and monitoring project was funded under the REMR Research Program, specifically the Floating Debris Control Systems project.

An electronic system using many components from a previous field study, Floating Breakwater Prototype Test-Monitoring Program (circa 1981-1983), obtained and stored data. After a limited analyzation, the system sent the data out from the inboom location via a radiofrequency (RF) transmitter to a receiver and modem at the dam (Figure 1). From the dam the data went by telephone line to the University of Washington and could also go to the District office or to other locations such as the US Army Cold Regions Research and Engineering Laboratory (CRREL).

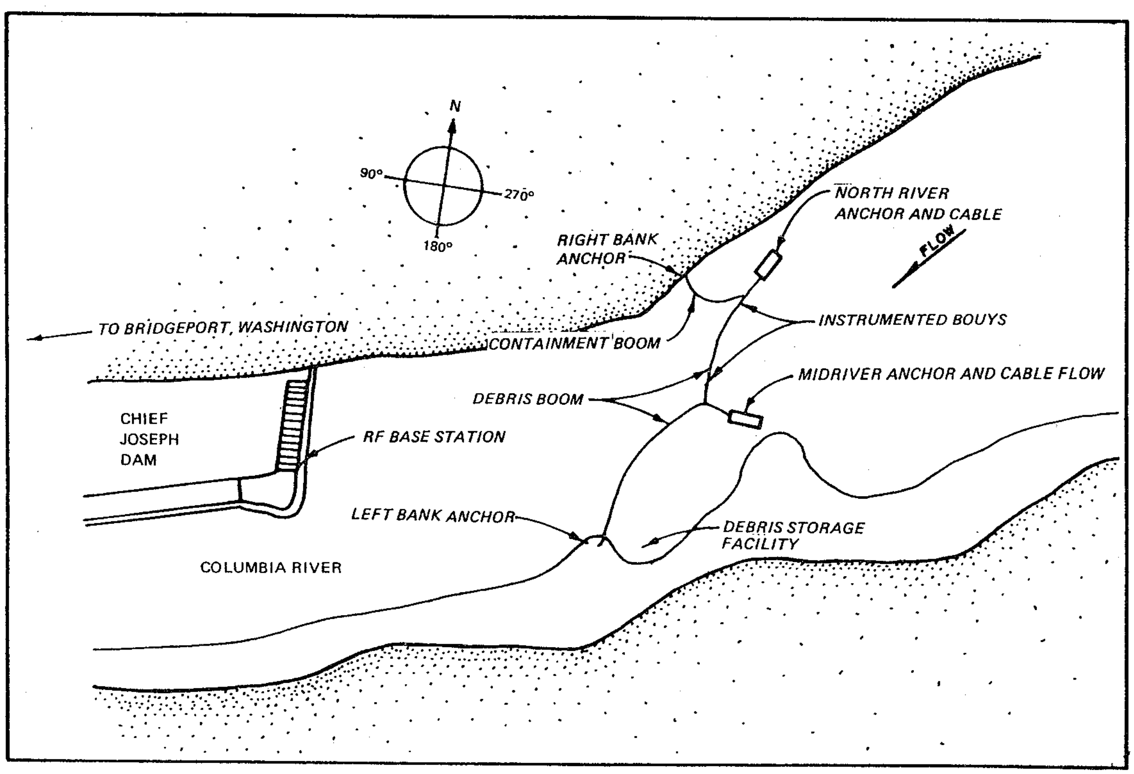


Figure 1. Location map and project layout.

Data were obtained for approximately 6 months at Chief Joseph Dam. The force levels registered were similar to those obtained previously at the downstream end. The data have been reduced by the University of Washington to the most important factors such as the wind and discharge effects and the sensitivity of the boom to waves. The force levels were found to be somewhat proportional to wind velocity and to the discharge rate through the dam intakes. The weather conditions during the whole instrumentation and monitoring project were very mild. There was very little activity to cause forces to develop: wind velocities were less than 35 miles per hr; maximum discharge was 180,000 cu ft per sec, but because of the large water depth in the pool, a water velocity of approximately 1.3 ft per sec was produced. Also, there was very little debris to push on the boom. Very low force levels mean that attempts to scale the effects to assumed higher levels are perilous. The University of Washington report by Christensen and Ratnayake* covers the most important items and is available upon request. It should be noted that the north reference for the wind direction in the field study was along the boom and, therefore, was approximately 45 degrees to true north.

A brief evaluation was made of some of the data using engineering-type wind and hydraulic drag relationships and assuming a parabolic shape for the shear boom sections. The boom structure has a sag-to-length ratio of approximately 1 to 22. Selected data from the report by Christensen and Ratnayake are shown in scatter plots (Figures 2 and 3). The data indicate that the boom is preloaded up to an average of 1,700

* D. R. Christensen and S. C. B. Ratnayake, 1988, "Results of the Field Data Collection and Analysis," prepared by the University of Washington for CRREL, Hanover, NH.

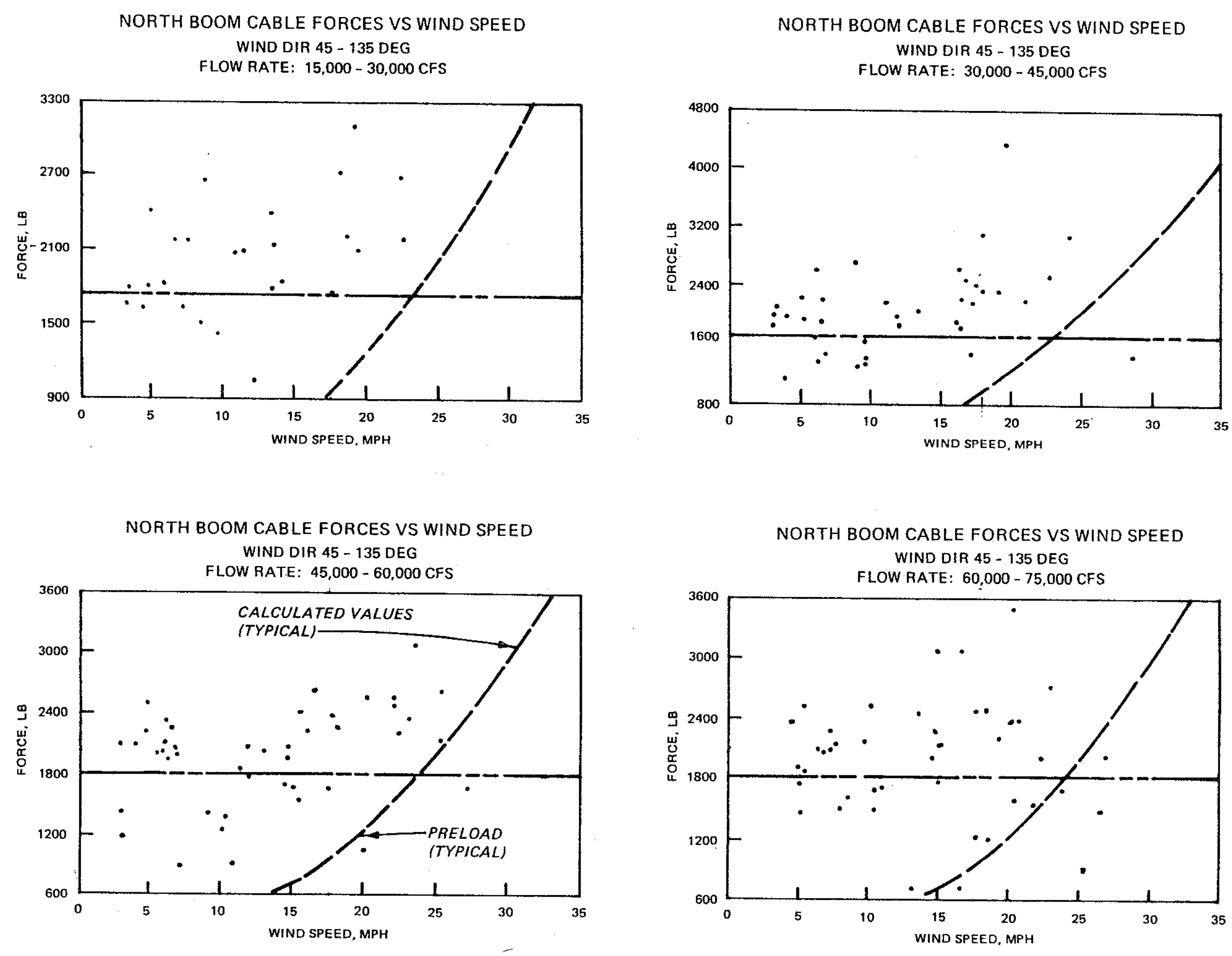


Figure 2. Debris boom force versus wind speed with various river discharges, Chief Joseph Dam, Columbia River.

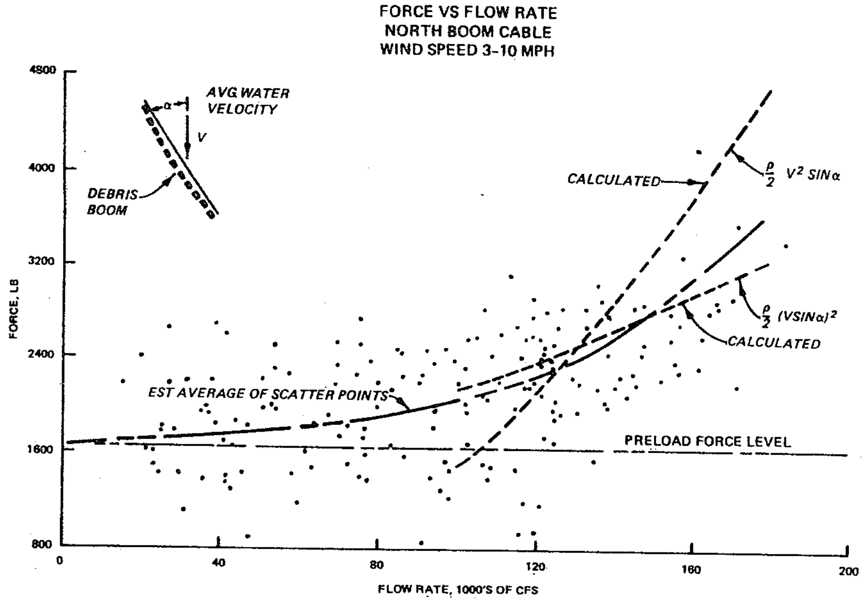


Figure 3. Debris boom force versus river discharge for low wind speeds, Chief Joseph Dam, Columbia River.

to 1,800 lb. However, 1,000 lb or so of this load is from the weight of the towplate and the midriver anchor cable connections. The preload of about 400 lb for the midriver anchor line is primarily the result of the weight of the cable and fittings between the boom and the float that supports the midriver anchor line.

The calculated values are plotted as dashed lines. These latter values are roughly at the same level as the data points, but the wind calculations (Figure 2) appear to be increasing in value more quickly with increases in velocity than do the field data. This fact is probably the result of the boom's being attached solidly only at one point, the left bank anchor. The submerged cofferdam anchors are solid, too, but their 600- or 700-ft-long anchor cables have intermediate floats and bends that give compliance to the boom connection points. Unless the driving forces are applied for a fairly long time, the whole boom will not respond as a unit to them. An average of the water drag scatter plot is drawn in Figure 3. These data seem more consistent with the theory, but unless a flood occurs, the forces from this source will not be much greater.

For debris boom improvement, it may be beneficial to have a few tons of preload in the cables to which the shear boom buoys are attached. The Seattle District engineering drawing E51-4-40 "Debris Boom North and Midriver Anchor Connections," dated January 31, 1979, shows four of the five floats on the north river anchor cable as being fully submerged, undoubtedly by preload. All of these floats, however, are actually out of the water; only the one nearest the anchor has a downward slope to it. A preload would reduce the number of loose oscillations that take place in the boom and would thereby reduce the tendency for wear in the connections and for bolts to loosen. This latter effect contributed to breaking the signal wires to the measuring unit on the north end of the debris boom. It should be noted that the left bank anchor point load sensor could be used to determine how much to shorten the cable for a particular preload.

It was unfortunate that the signal wire from the load cell in the north river anchor connection was damaged before data were obtained from it. This connection is free to move around within the bounds of its attachment to the river end of the containment boom and to the float-supported north river anchor cable. The anchor cable reaches down to its anchor about 150 ft underwater. Any analysis of the structure would have been helped if the loads at the north end could have been measured.

For further information, contact Roscoe E. Perham at (603) 646-4309.

Russ Perham is a mechanical engineer in the Ice Engineering Research Branch, Experimental Engineering Division, US Army Cold Regions Research and Engineering Laboratory, Hanover, New Hampshire, and has been with the laboratory since 1962 working in various areas including ice control. He received a B.S. degree in mechanical engineering from the University of Maine and an M.S. degree from Rensselaer Polytechnic Institute and has taken advanced courses at Dartmouth College. He is principal investigator for REMR Work Unit 32320, "Floating Debris Control Systems."

Comparison of Corps of Engineers' and US Bureau of Reclamation's Methods for Calculating Uplift Pressures

by
Carl Pace
US Army Engineer Waterways Experiment Station

Stability analyses are performed to determine base pressures and the resistance of a structure to sliding and overturning. Calculating uplift pressures is one step in the performance of a stability analysis for a dam or other concrete structure. The Corps of Engineers and the US Bureau of Reclamation (USBR) use different procedures for calculating uplift pressures, but the results obtained are the same if the correct USBR procedure is followed. The USBR has a simplified method that is not applicable to all situations and that if used indiscriminately, can yield erroneous results.

A study was done to explain the differences between the Corps' method and the USBR's simplified method for evaluating the stability of a structure on rock foundations and to discuss the significance of such differences.

In both the Corps' and the USBR's analyses, the uplift pressure is considered to vary linearly along the seepage path from headwater to tailwater except as modified by drain efficiency. Both methods calculate the base pressures with the general formula

$$\sigma_z = \frac{P_z}{A} \pm \frac{M_{xx}C_y}{I_{xx}} \pm \frac{M_{yy}C_x}{I_{yy}}$$

where

σ_z = normal stress at base-foundation interface

P_z = force normal to base-foundation interface

A = area of base that is in compression

M_{xx}, M_{yy} = moment about x-x and y-y axes, respectively

C_y, C_x = coordinates to position where normal stress is calculated

I_{xx}, I_{yy} = moment of inertia about x-x and y-y axes, respectively

Two overall uplift conditions can occur in a stability analysis: the structure cannot have drain efficiency or it can have drain efficiency. If the structure does not have a drain, it can be totally compressive against the foundation, or a portion of the base cannot be compressive at the base-foundation interface.

WITHOUT DRAINS

If a structure does not have a drain and a portion of the base is not in compression, the Corps assumes that a crack exists at the concrete-foundation interface for the noncompressive portion of the base. This condition causes full uplift to be applied under the noncompressive part of the base, resulting in more overturning force than was assumed in the previous computation and thereby increasing the noncompressive area of the base. The above method results in the Corps' using an iterative solution to determine the noncompressive area of the structure's base. The iterative solution continues until the calculated crack length is the same as that assumed, making the resultant of the applied forces colinear with the resultant of the vertical base pressure diagram. In this case, the simplified method for calculating uplift used by the USBR produces the same results as the Corps' method.

However, if a structure has no drains and the total base is in compression, the USBR's simplified method does not give the correct results. The USBR's equations (see "Design of Gravity Dams," United States Department of the Interior, Bureau of Reclamation, 1976 ed., pp 32 and 33) are derived for a cracked-base situation (i.e., a portion of the base is not in compression). The shape used in the USBR's derivation uses B5-A3 (Figure 1a) as one leg of a right triangle. The base pressure diagram for no drains and the total base in compression (Figure 1b) shows the actual B5-A3 dimension is one side of a trapezoid. The USBR's simplified equations are not applicable to the case in which the total base is in compression.

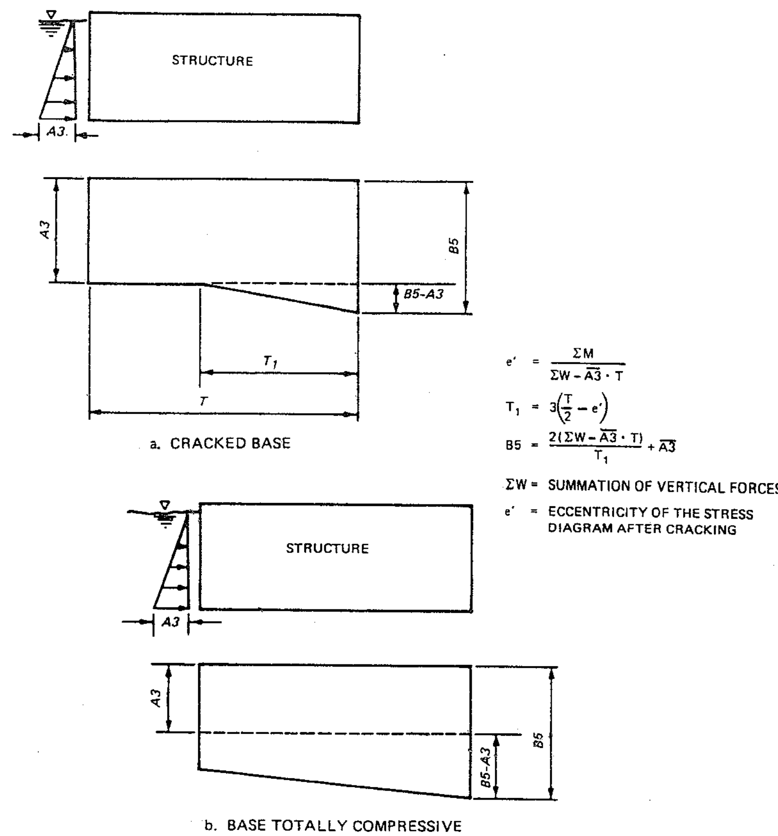


Figure 1. Illustration of why the USBR's simplified equations developed for a cracked base are not applicable when the total base is in compression.

WITH DRAINS

Another category of possibilities is a structure with a drain. The structure with a drain can have the base in compression, or it can have a portion of its base not in compression. If the total base is in compression, the USBR's simplified equations are not applicable because they are for a crack at the base-foundation interface, and also they do not allow for drain effectiveness. The solutions by the USBR's simplified equations and the Corps' methods are not comparable.

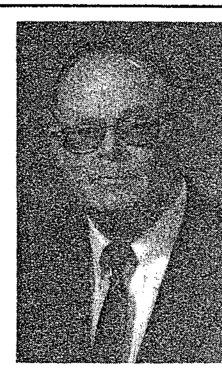
If the base is cracked and the crack does not pass the drain, the USBR's simplified equations and the Corps' methods will not produce the same results; the Corps uses drain effectiveness until the crack passes the drain. The Corps' method and the USBR's simplified equations give the same results when the crack at the base-foundation interface is past the drain.

CONCLUSIONS

The method used by the Corps and the USBR's method for calculating uplift pressures under concrete monoliths on rock foundations produce the same results. However the USBR's simplified method is applicable only for the conditions in which a structure has no drain and has a cracked base or for a structure with a drain where the noncompressive portion of the base extends past the drain and has no drain effectiveness. The USBR's simplified equations must be used only for these cases.

For further information, contact Carl Pace at (601) 634-3221.

Carl Pace is a research civil engineer in the Concrete Technology Division, Structures Laboratory, Waterways Experiment Station. He received an M.S. degree in engineering mechanics from Mississippi State University and a Ph.D. in engineering from the University of Arkansas. He has conducted a wide range of studies in the maintenance and preservation of concrete structures.



REQUEST FOR ARTICLES

The REMR Bulletin is actively soliciting articles in any of the areas being addressed by the REMR Research Program. Articles by individuals outside the Corps will be considered if relevant to REMR activities of the Corps.

To submit an article, write to: Commander and Director, US Army Engineer Waterways Experiment Station, ATTN: CEWES-SC-A, PO Box 631, Vicksburg, MS 39180-0631. Guidelines are available upon request.

When submitting photographs with articles, please provide glossy prints or film rather than prescreened negatives.

FIELD REVIEW GROUP MEETINGS

The Eleventh REMR Field Review Group (FRG) Meeting was held in San Francisco, California, April 12-14, 1988. The meeting was sponsored by the South Pacific Division. Progress in the work units of the seven problem areas of the REMR Research Program was discussed.

The Twelfth Meeting of the FRG is scheduled for August 30 - September 1, 1988, in Vicksburg, Mississippi, and will be sponsored by the US Army Engineer Waterways Experiment Station. More information regarding this meeting may be obtained by calling Lee Byrne, (601) 634-2587, or by writing to: Commander and Director, US Army Engineer Waterways Experiment Station, ATTN: CEWES-SC-A, PO Box 631, Vicksburg, MS 39180-0631.

VIDEO REPORT ON PRECAST CONCRETE STAY-IN-PLACE FORMING SYSTEM NOW AVAILABLE

REMR Video Report CS-87-1, "A Precast Concrete Stay-in-Place Forming System for Lock Wall Rehabilitation," is now available to be checked out from the Information Technology Laboratory, Waterways Experiment Station (WES).

The 20-min video report summarizes a new method of minimizing cracking in lock wall resurfacing through the use of precast panels as stay-in-place forms. A precast panel rehabilitation system was designed by ABAM Engineers, Inc., in Phase I of a contract with WES. Phase II was a constructibility demonstration in which eight panels were precast in Colorado and erected on two one-half scale simulated lock wall monoliths at WES. The purpose of the demonstration was to evaluate the feasibility of the stay-in-place forming system without the risk and investment of undertaking a full-scale lock rehabilitation.

To borrow a copy of REMR Video Report CS-87-1, contact the Library of the Information Technology Laboratory, CEWES-IM-TL-R, (601) 634-4120.

NEWS IN BRIEF

Robert E. Pletka has been appointed as the Technical Monitor for the Electrical and Mechanical Problem Area. He is very familiar with the REMR Program as he was an original member of the REMR Field Review Group.

COVER PHOTOS

Anchor embedment in concrete with vinylester resin.

Aerial view, looking downstream at the 3,000-ft-long debris boom upstream of Chief Joseph Dam on the Columbia River, Washington.

The REMR *Bulletin*

The REMR Bulletin is published in accordance with AR 310-2 as one of the information exchange functions of the Corps of Engineers. It is primarily intended to be a forum whereby information on repair, evaluation, maintenance, and rehabilitation work done or managed by Corps field offices can be rapidly and widely disseminated to other Corps offices, other US Government agencies, and the engineering community in general. Contributions of articles, news, reviews, notices, and other pertinent types of information are solicited from all sources and will be considered for publication so long as they are relevant to REMR activities. Special consideration will be given to reports of Corps field experience in repair and maintenance of civil works projects. In considering the application of technology described herein, the reader should note that the purpose of *The REMR Bulletin* is information exchange and not the promulgation of Corps policy; thus, guidance on recommended practice in any given area should be sought through appropriate channels or in other documents. The contents of this bulletin are not to be used for advertising, publication, or promotional purposes. Citation of trade names does not constitute an official endorsement or approval of the use of such commercial products. *The REMR Bulletin* will be issued on an irregular basis as dictated by the quantity and importance of information available for dissemination. Communications are welcomed and should be made by writing the Commander and Director, US Army Engineer Waterways Experiment Station, ATTN: Lee Byrne (CEWES-SC-A), PO Box 631, Vicksburg, MS 39180-0631, or calling 601-634-2587.

DWAYNE G. LEE
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